<u>Structural Implication of Post-Tensioned Ducts in Prestressed Concrete Girders</u>

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1 Introduction

Prestressed concrete girders are used extensively to support bridge decks in highway bridges. The maximum length of a typical prestressed concrete girder is generally limited to what can be transported safely by trucks to the construction site – approximately 160 feet. With the use of post-tensioned strands placed in ducts cast into the girders, multiple pretensioned concrete girders can be spliced together to form a continuous member along the length of a bridge, allowing greater structural continuity and more efficient use of materials. However, due to the location of the ducts in the relatively thin web regions of the girders, new failure modes can control the shear strength of the members. The structural implications of corrugated steel and plastic ducts cast in the web regions of spliced Tx-Girders under the effects of compressive loading are studied in this report. Spliced girder technology is detailed in chapter 2. A literature review of current U.S. provisions for design of spliced girders and how they are modeled is provided in Chapter 3. The experimental program is described in chapter 4. The experimental results and conclusions are discussed in chapters 5 and 6 respectively.

2 Spliced Girder Technology

2.1 Basic Description

Splicing is essentially a method whereby girders that would normally be too heavy or large to transport may be created on-site from precast segments. Once spliced together using cast-in-place concrete, the structure is generally post-tensioned to improve the structural performance of the spliced regions (Castrodale & White, 5). Since their first use in North Africa in 1944, over 250 spliced girder bridges have been erected. In the United States, splicing technology has been used most commonly in Florida, Washington, Oregon, and Colorado, though a fair number of bridges have been built in other states (Castrodale & White, 17).

2.2 Comparison to Segmental Box Girders

Spliced girder construction is essentially similar to segmental box girder construction, but several distinct differences set them apart. Both methods use post-tensioned strands in grouted ducts to create continuous members from individual precast pieces. However, precast segments in spliced girder construction tend to be much longer compared to the overall span length, are generally I- or U-shaped, and are referred to as "girder segments." Also, segmental box girders are generally spliced using match casting, whereby the surfaces to be spliced in the field are cast with shear keys designed to fit together. This method is less preferable to cast-in-place splicing when using girder segments due to opposing angles in the end regions induced by camber resulting from eccentric pretensioning force in the individual girder segments. Box girders also tend to have an integral full width deck, whereas spliced girders are topped with a cast-in-place deck after erection (Castrodale & White, 10).

2.3 Benefits of Spliced Girder Technology

When used properly, the spliced girder method can increase span lengths, reduce overall structural depth and increase construction speeds, in addition to improving overall aesthetics. Longer spans can allow designers to reduce the amount of supporting piers, thus diminishing material costs, increasing driver safety, and helping to avoid obstacles or waterways beneath the structure. Use of precast segments also decreases the amount of falsework and on-site workers required, effectively reducing construction time and costs (Castrodale & White, 5).

2.4 Applications

There are several reasons why designers may choose to use spliced girders for a given project whether it is a simple or continuous span bridge. If a simple span is to be used, spliced girders are often more economical when full-length precast girders would be difficult to erect or be transported to the site. In locations where roads leading to the construction site would be unable to support the weight or length of trucks carrying a full-length girder, breaking the girders into smaller pieces to be assembled on-site may allow the use of precast concrete girders. Contractors may choose to use spliced girder segments to allow use of a smaller, more economical crane during construction, or if locally available equipment would be unable to lift a full-length girder into place. The program requirements of some situations, such as single-point urban interchanges, call for a minimized girder depth. Use of spliced girder technology can alleviate transportation problems of such girders caused by the extensive length and added weight caused by decreasing the depth. Continuous span bridges with intermediate supports are the most typical application of spliced girders. These include full-span girders, which are joined above interior supports; partial span girders, which are spliced both between and at interior supports; and haunched girders (Castrodale & White, 6). Haunched girders have varying structural depths that expand near supporting piers by increasing the height of either the web region or bottom flange of the member. Due to the immense height and weight of such members, they are generally only feasible above navigable waterways where they can be assembled directly from a barge (Castrodale & White, 37).

3 Literature Review

3.1 Shear Strength of Post-Tensioned Members

3.1.1 Shear Strength Modeling

Strut and tie modeling is a method of determining the strength of a reinforced concrete member by treating the beam as a truss consisting of series of concrete struts, which transmit compressive forces; and steel ties, which transmit tensile forces. Shear strength is computed by disregarding the tensile strength of concrete, and assuming compressive forces will be carried by concrete struts oriented at 45° to the longitudinal axis, which are resisted by the vertical steel stirrups loaded in tension. It has been shown that truss models which assume an angle of 45° are very conservative, but are typically precise enough for design purposes (445R, 5).

Compression field theory (CFT) is a more advanced form of strut and tie modeling which takes into account the actual angle at which the diagonal struts form. The angle can be derived from Mohr's circle of strain, and is a function of the ratio of steel reinforcement to the area of concrete in both the longitudinal and transverse directions (445R, 6).

Modified compression field theory (MCFT) is a further refinement of strut and tie modeling which takes advantage of cracked concrete's limited tensile strength. The method also takes into consideration the possibility of failure due to high localized stresses at crack locations, as well as the influence of shear forces on the longitudinal reinforcement (445R, 9). Though MCFT can predict the actual strength of a given section to a high level of precision, the procedure is generally viewed to be too tedious for typical design work.

3.1.2 Effects of Post-Tensioning Ducts on Shear Strength

The introduction of a post-tensioning duct into the web region of a girder creates an area of tensile stress above and below the duct due to the divergence of compressive stress flow around the duct. These tensile regions typically cause small cracks to form at the top and bottom of the duct, which advance outward from the duct as load is increased, resulting in a splitting failure of the specimen (Muttoni, 729). The reduced strength of the shear section is typically modeled in one of two ways: a reduced effective width of web, or a reduced compressive strength of the concrete itself (Muttoni, 308).

3.1.3 Relevant Parameters

The main parameters affecting the shear strength of post-tensioned girders are the duct material, the stiffness of the grout, whether or not the duct is grouted, and the ratio of duct width to the overall web thickness. Post tensioning ducts are commonly available in steel or high-density polyethylene (HDPE) plastic. Plastic ducts are less costly to manufacture, but result in a lower strength than steel ducts because of the lack of bond at the concrete-plastic interface. The stiffness of grout used relative to the stiffness of the concrete also affects the strength of the section. If the stiffness of the grout is much lower than that of the surrounding concrete, more stress will tend to flow through the surrounding concrete, increasing the resulting

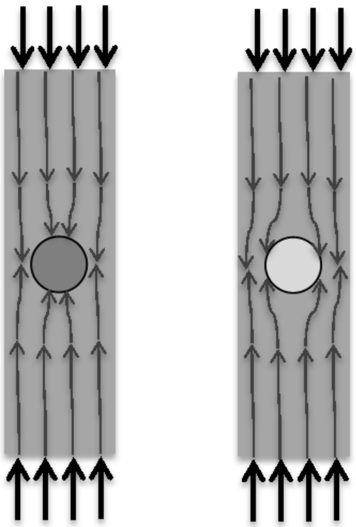


Figure 1: Comparison of Stiff Grout (a) and Soft Grout (b)

tensile stress above and below the duct (Figure 1a). If the stiffness of the grout is greater than that of the surrounding concrete, more stress will tend to flow through duct itself, creating tensile stress farther away from the location of the duct (Figure 1b). Ducts with grout strength most like the surrounding concrete will have the greatest strength, because stress is carried more uniformly across the thickness of the web. Similarly, ungrouted ducts greatly reduce the performance of the section, because no stress can be carried through the open duct region.

3.1.4 Need for HDPE Duct Data

Currently, there is very little data on the effects of plastic ducts on shear strength of concrete girders. Because concrete binds to steel while curing, some stress can be transferred through the duct rather than through the surrounding concrete, thus increasing the strength of the section. However, concrete will not bind to high-density polyethylene, and therefore no stress can be transferred at the concrete-HDPE interface. Because of this, sections with HDPE and steel ducts will have different strength characteristics, as confirmed by previous testing (Muttoni, 731). Current code provisions do not distinguish between steel and plastic ducts in strength reduction calculations.

3.2 Results from Literature

3.2.1 Current Code Provisions

In the United States, the principal codes for reinforced and precast concrete structures are AASHTO, which governs the construction of highway bridges, and ACI 318, which governs concrete building design. The 2010 edition of AASTHO has two provisions for the calculation of shear strength of members with imbedded post-tensioning ducts: section 5.8.6.1 for segmental construction, and section 5.8.2.9 for general construction. ACI currently has no provisions for the effect of ducts on shear capacity. In AASHTO LRFD, the reduction factors are implemented by use of a web-width reduction factor of the form:

$$\eta_d = 1 - k \cdot \delta \qquad \textbf{Equation 1}$$

where δ is the inner duct diameter to web thickness ratio, and k is a factor to account for loss of strength due to duct presence. For general construction, k is equal to ½ for ungrouted ducts and ¼ for grouted ducts. For segmental construction, k is equal to 1 for ungrouted ducts and ½ for grouted ducts. The segmental k factors are higher due to a lack of redundancy in segmental box girder construction, which calls for a higher factor of safety.

3.2.2 Panel Tests by Muttoni

Previous tests were conducted by Aurelio Muttoni et al. at Ecole Polytechnique Federale de Lausanne in Switzerland. The tests compared strength of a series of 12 panels cast in the laboratory and 4 panels extracted from an actual bridge girder in the field. The laboratory panels included specimens with both HDPE and steel ducts, some of which were injected with grout and some left empty. These were compared to two panels with no duct present, which served as controls (Muttoni, 734).

4 Experimental Program

4.1 Specimen Design

Because of the large costs associated with full-scale beam testing, full-scale web tests were conducted in order to determine the strength characteristics of web sections with steel and plastic ducts compared to that of a normal web. Full-scale web specimens consist of 24” square panels of varying width to represent web regions of different thicknesses. All panels contain #4 deformed bars as represented in Figure . Specimens with steel or plastic ducts cast horizontally in the center of the panel were compared to control specimens containing no ducts.

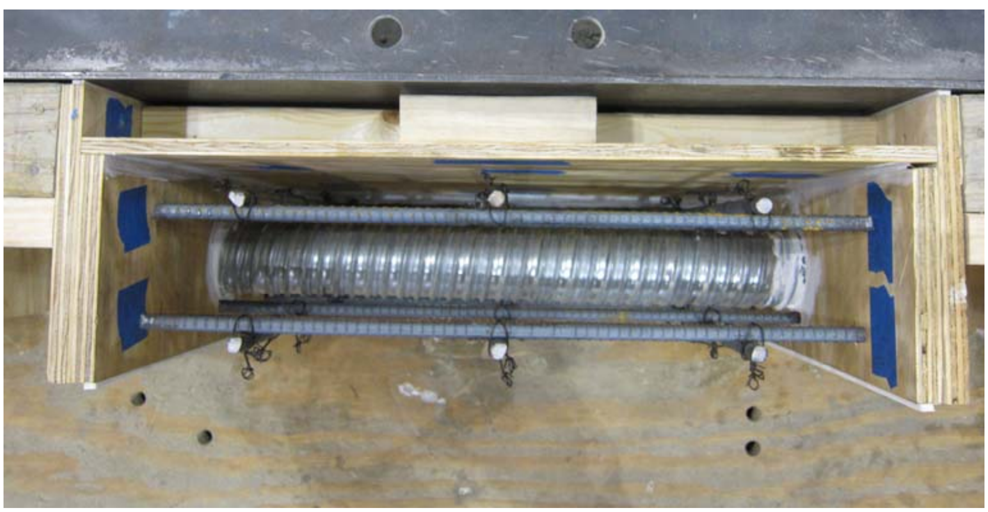


Figure 2: Typical Panel Reinforcement

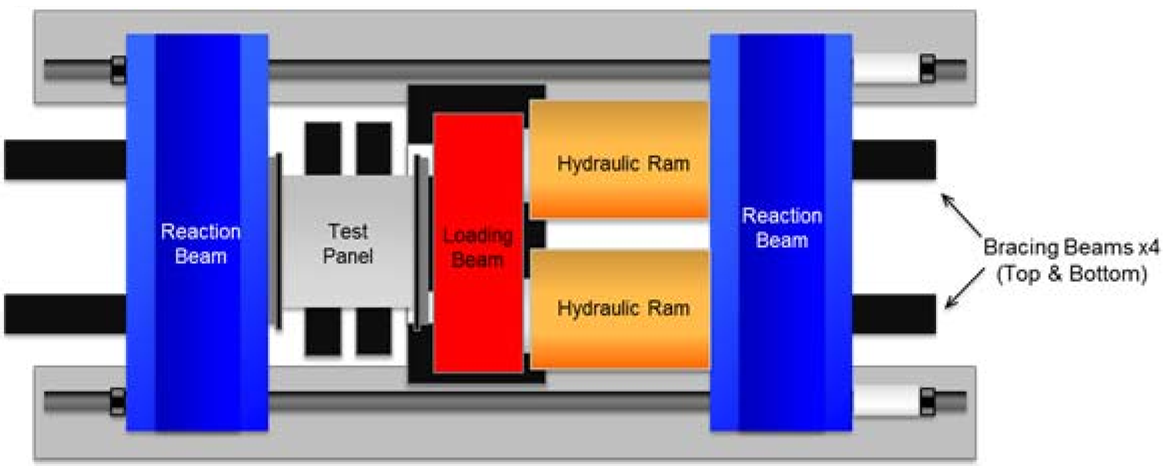
4.2 Casting

Panels were cast at Ferguson Structural Engineering Lab using 30-foot long side forms bolted down to two layers of 3/4-inch plywood supported by 4x4 lumber spaced at regular intervals. Partition forms held the ducts in place during casting separated individual panels as shown in Figure 3. Reinforcement was held into place by tying them to #2 bars, which were set into the partition forms. Once the concrete had hardened, panels were removed from the forms and set aside for further curing. After roughly two weeks, the panels were positioned with the duct oriented vertically and filled with grout and 0.5 inch strands to represent post-tensioning steel. The number of strands differed with each size of duct, with 7 strands in 2.5 inch ducts, 9 strands in 3 inch ducts, and 12 strands in 3-5/8 inch ducts. The strands were only placed to augment the stiffness of the duct region, and were not stressed during grout placement.

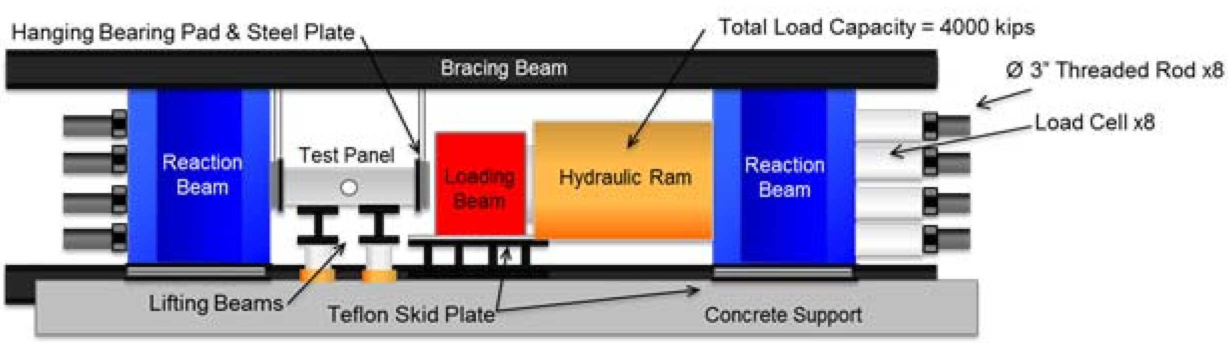
Figure 3: Panel Forms

4.3 Test Setup

Panel tests were performed using the test setup shown in Figure 4. The resisting frame consisted of two large beams salvaged from a prior project, which were restrained by 8 steel rods 3 inches in diameter. Two hydraulic rams with a combined capacity of 4 million pounds were used to create compressive force, which was transmitted to the panel using a high stiffness transfer beam. The transfer beam was allowed to slide by attaching 4” by 6” pieces of Teflon to the underside of the transfer beam, which were supported by two pieces of Teflon glued to the top of the track beam.



(a) Top View



(b) Side View

Figure 2: Testing Frame

4.4 Variables Tested

4.4.1 Grouted Versus Ungrouted Ducts

Ducts are generally grouted to prevent the intrusion of water into the tendon, which could cause corrosive damage in strands. Before grout is placed, the large void in the web section created by the duct greatly reduces the strength of the member. By testing both grouted and ungrouted specimens, their strength can be compared to that of an unreduced section, i.e. one with no duct.

4.4.2 Plastic Versus Steel Ducts

Because concrete is able to form a bond with a steel duct, some load is able to be transferred into the duct when grout is present. HDPE ducts do not form a bond with the concrete and therefore have different strength characteristics. Current codes do not distinguish between them.

4.4.3 Size Effects

Varying thicknesses of panels were tested to explore the effects of increasing the web thickness on structural performance. The webs of bulb tee girders can easily be made wider by increasing the distance between side forms during casting. The effects of web width on overall strength were compared directly by casting 5, 7 and 9 inch panels in one set.

5 Results

5.1 Ungrouted Ducts

When no grout is present in the duct, the strength of the section was found to be insignificantly affected by the duct material. Figure 5 is key figure which explains the plots given in figures 6 through 9. Results from both data collected at Ferguson Structural Engineering Lab (Appendix A) and previous research are plotted against the reduction factor that would be given by ACI and AASHTO provisions, which were discussed in Section 3.2.1. Any data points that fall below the lines representing the various code equations would be designed unconservatively, i.e. their modeled strength would be greater than that which will be actually attained in the field.

As can be seen in Figure 6, neither the general or segmental provisions for ungrouted ducts adequately account for the reduction in strength due to an open void being present in the web region of a girder. Because a vast majority of the data points have an actual strength below what the code would apply to the section, any beam designed using those provisions would be designed unconservatively.

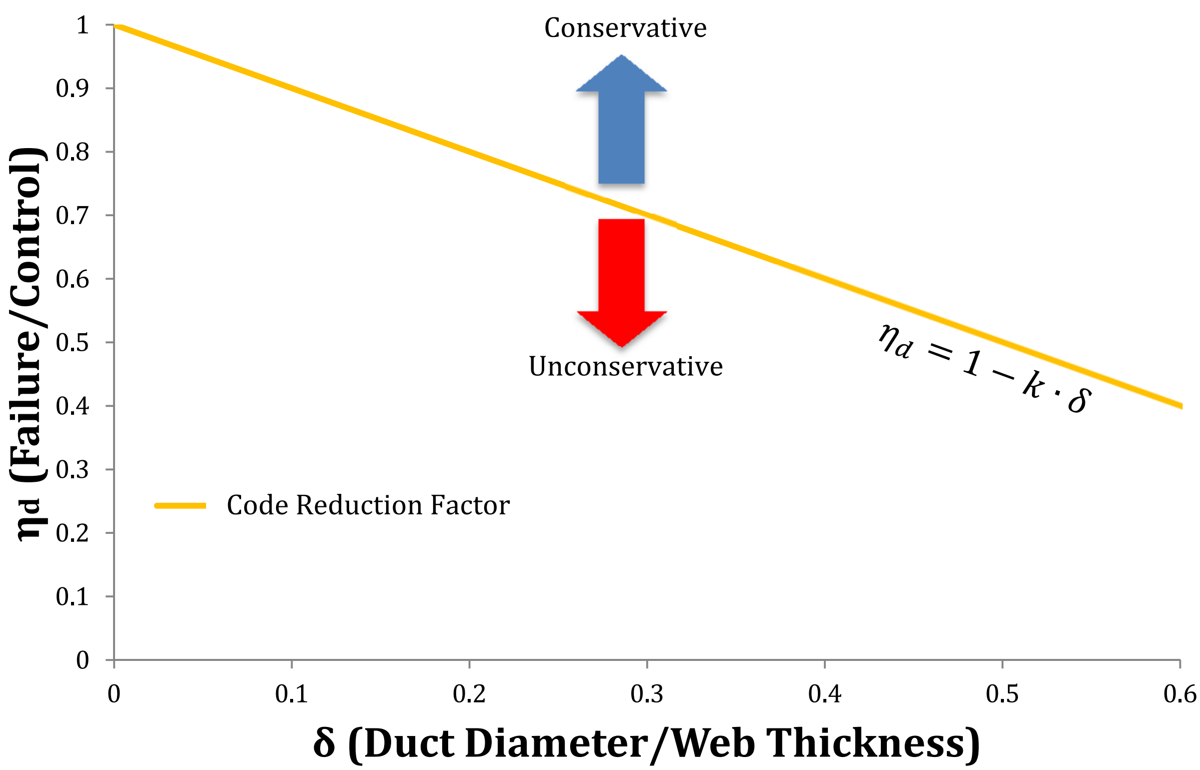


Figure 3: Key Figure

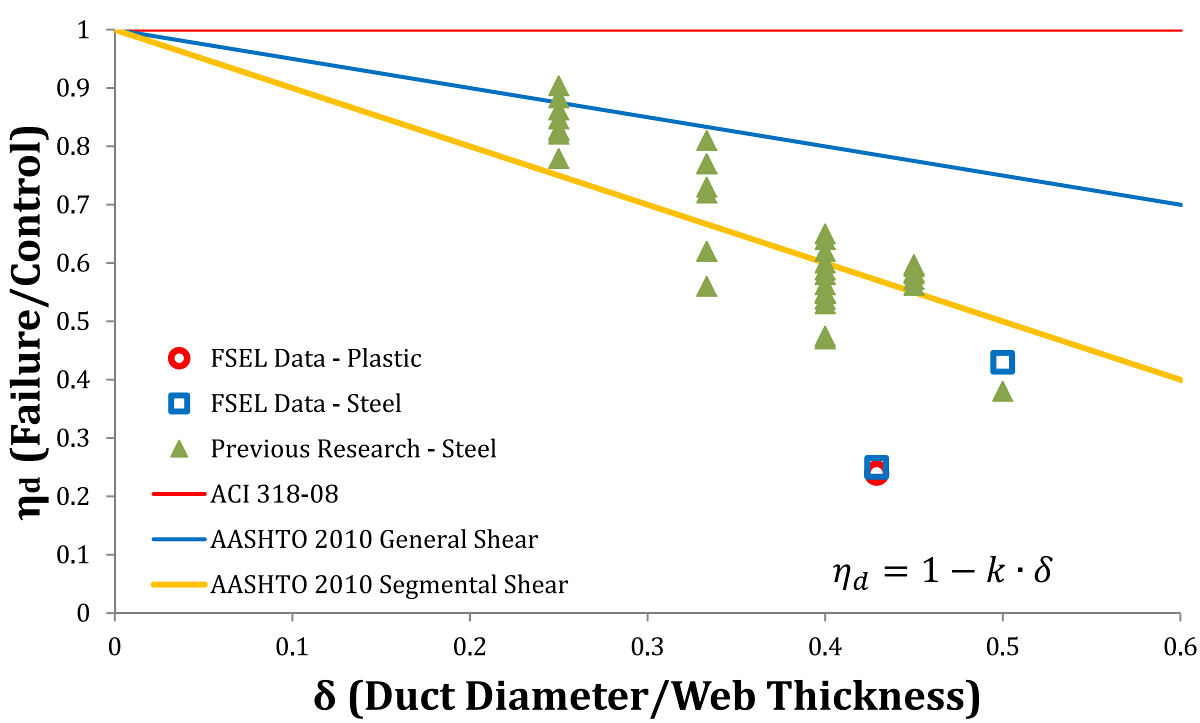


Figure 4: Ungrouted Duct Results

5.2 Steel Ducts

Though steel ducts were shown to perform better than plastic ducts, current reduction factors still may not adequately account for the strength reduction caused by their presence. The vast majority of previous research data was conducted using steel ducts. Our results generally agreed with these data, with the exception of a few outliers, not included on the plot, which may have had defects within the panels or non-uniform loading conditions. These results indicate that current reduction factors may not be adequate to accurately predict the shear strength of girders with post-tensioned ducts.

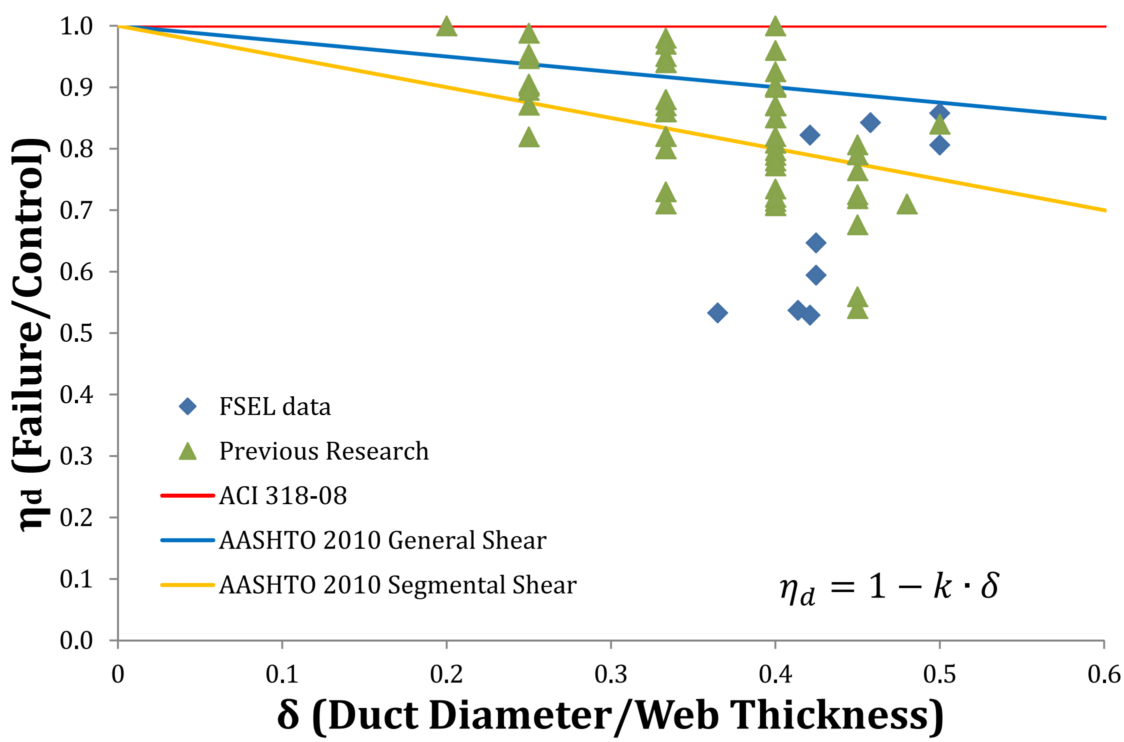


Figure 5: Steel Duct Results

5.3 Plastic Ducts

Prior to our testing, very little laboratory data was available on the effects of plastic ducts. Given in Figure 7 is our experimental data as well as data from previous research. Because current codes do not account for the difference between steel and plastic ducts, their strength is not accurately modeled by current code provisions. As expected, our comparison of steel ducts to plastic ducts, given in Figure 8, found that though the duct material had little effect on the strength when left ungrouted, ducts which were grouted had significantly higher strengths with steel ducts than plastic ducts. This is most likely because concrete can form a chemical bond with the steel duct as it cures, allowing a mechanism to transmit some of the load from the surrounding concrete into the duct region. When a HDPE duct is used, the concrete cannot form a bond with the duct, and therefore can transfer much less load through the grout inside the duct. In order to model this behavior more accurately, different factors should be implemented to distinguish between plastic and steel ducts.

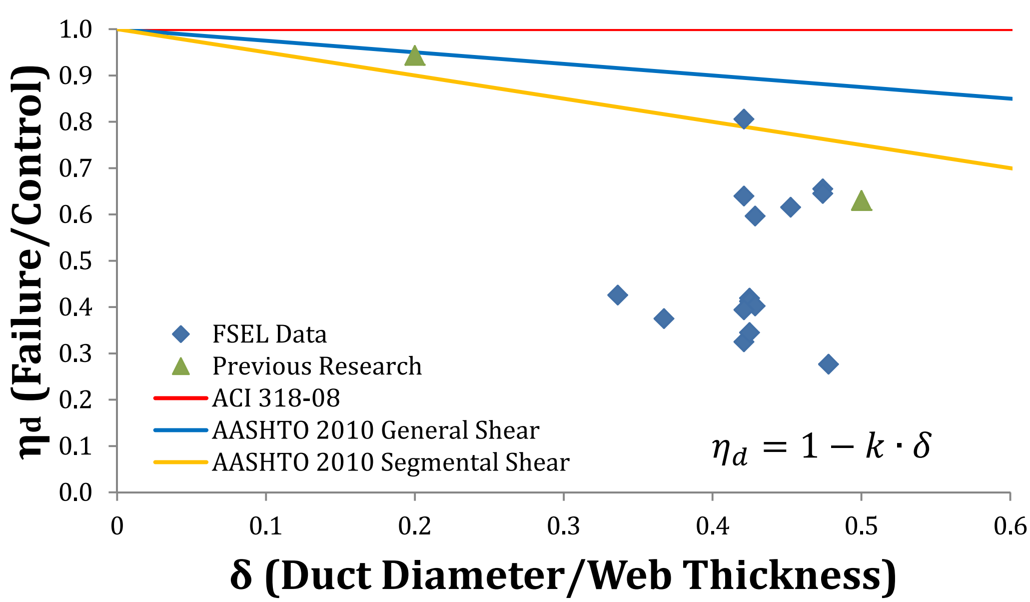


Figure 7: Plastic Duct Results

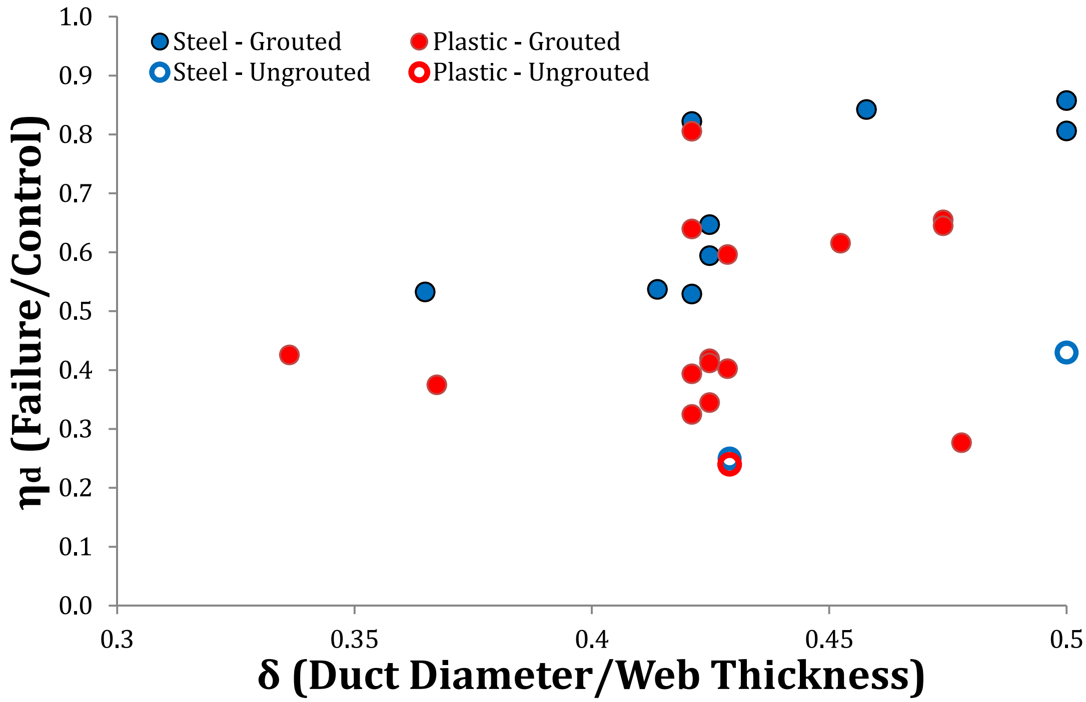


Figure 8: Comparison of Plastic and Steel Duct Data

5.4 Size Effects

In one specific set of panels, 5, 7, and 9 inch panels were directly compared against each other using both plastic and steel ducts. Though designers using current code provisions would predict an increase in relative strength as the duct diameter to web-thickness ratio is decreased, our results, given in Figure 9, show that when the web thickness is increased, the relative strength actually decreased. This is because current code equations use a reduced web width to account for the presence of a duct, which does not take into account the different failure modes of web regions with and without ducts.

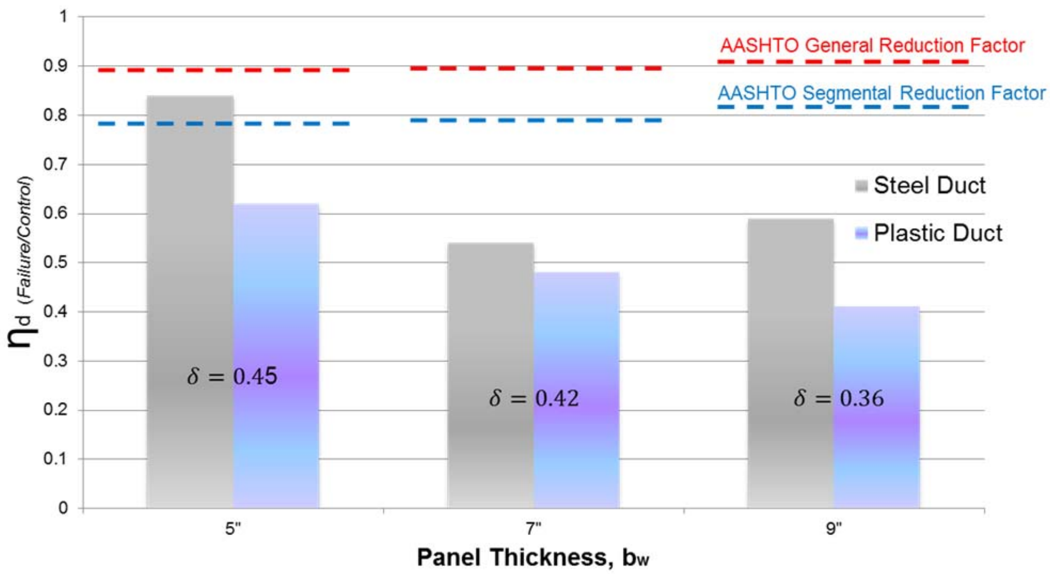


Figure 9: Direct Comparison of 5, 7 and 9 Inch Panel Data

When there is a duct present in the member, the failure mode is a splitting failure which is a function of the tensile strength of concrete and the effective height of the shear region as shown in Figure 10. When no duct is present, the failure mode is a crushing type failure, which is governed by the compressive strength of the concrete and the width of the web region. These failure modes are not linearly dependent on one another, and therefore a reduced web width is not an effective way of modeling the reduction in strength due to the presence of a duct. Future code provisions should therefore have a different form, which takes into account the tensile failure mode.

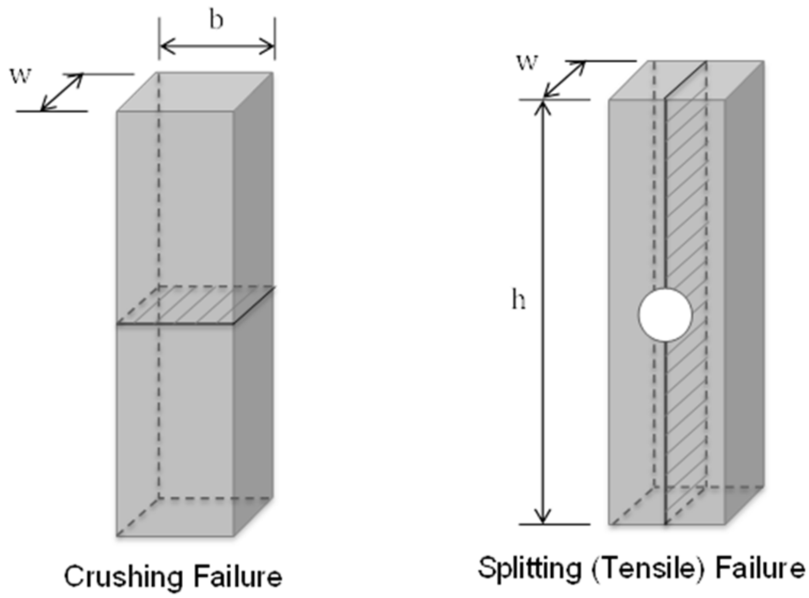


Figure 10: Comparison of Crushing and Splitting Failure Modes

6 Conclusions

More information on the behavior of post-tensioned beams will be required as spliced girder bridges become more prevalent in the United States. Current code equations may be unconservative, and need to be recalibrated to accurately depict the strength of members with increased web thicknesses, as well as beams which use plastic ducts rather than steel. It was shown that the material which the duct is made from does not affect significantly strength without presence of grout. Further testing is currently being conducted at Ferguson Structural Engineering Lab to correlate data collected from full-scale panel tests to full-scale beams. These data will be used to formulate new equations to be suggested to organizations who dictate building codes such as the ACI and AASHTO.

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Appendix A: Tabulated Results

Set and Specimen Number	Actual Specimen Width (in)	Actual Specimen Thickness (in)	Concrete f'c (ksi)	Duct Type	Grout Presence	Inner Duct Diam. (in)	Diameter to Thickness Ratio	Failure Load (kips)	Failure Load Normalized Against Control (ηd)
Set1: P5	24	5	6.23	Contr		0	0	625.2	100%
Set1: P7	24	5	6.23	Plastic	Grouted	2.37	0.474	409.6	66%
Set1: P9	24	5	6.23	Plastic	Grouted	2.37	0.474	403.3	65%
Set1: P4	24	5	6.23	Steel	Ungroute	2.5	0.5	267.7	43%
Set1: P6	24	5	6.23	Steel	Grouted	2.5	0.5	504.0	81%
Set1: P8	24	5	6.23	Steel	Grouted	2.5	0.5	536.3	86%
Set3: P5	23.9375	7	9.39	Contr		0	0	1192.7	100%
Set3: P1	24	7.0625	9.39	Plastic	Grouted	3	0.424	506.0	42%
Set3: P2	24.125	7.0625	9.39	Plastic	Grouted	3	0.425	499.9	41%
Set3: P3	24.125	7	9.39	Plastic	Ungroute	3	0.428	294.2	24%
Set3: P6	24.0625	7	9.39	Plastic	Grouted	3	0.428	482.4	40%
Set3: P4	23.9375	7	9.39	Steel	Ungroute	3	0.428	298.8	25%
Set3: P7	23.9375	7.0625	9.39	Steel	Grouted	3	0.425	778.6	65%
Set3: P8	24.125	7.0625	9.39	Steel	Grouted	3	0.424	720.7	59%
Set4: P1	23.875	7.125	8.17	Contr		0	0	1016.9	100%
Set4: P2	24	7.125	8.6	Contr		0	0.000	1143.5	100%
Set4: P5	23.9375	7.125	8.17	Plastic	Grouted	3	0.421	401.6	39%
Set4: P6	24.0625	7.125	8.6	Steel	Grouted	3	0.421	606.9	53%
Set5: P1	23.75	7	3.62	Contr		0	0	515.4	103%
Set5: P2	24	7.0625	3.62	Contr		0	0.000	493.9	97%
Set5: P4	24.125	7	3.62	Plastic	Grouted	3	0.428	302.8	60%
Set5: P6	24	7.125	3.62	Plastic	Grouted	3	0.421	328.9	64%
Set5: P8	24	7.125	3.62	Plastic	Grouted	3	0.421	414.3	81%
Set5: P5	24	7.125	3.62	Steel	Grouted	3	0.421	422.8	82%
Set5: P7	24	7.125	3.62	Steel	Grouted	3	0.421	524.9	102%
Set5: P9	24	7.125	3.62	Steel	Grouted	3	0.421	560.5	109%
Set7: P1	23.9375	7	10.15	Contr		3	0.428	1217.0	100%
Set7: P2	23.9375	7	10.62	Contr		3	0.429	1219.3	100%
Set7: P8	24.1875	7.0625	10.62	Plastic	Grouted	2 3/8	0.336	529.4	43%
Set8: P1	23.875	7.125	11.16	Contr		0	0	1643.4	100%
Set8: P2	23.875	7	11.16	Contr		0	0.000	1653.7	100%
Set8: P3	24.3125	7.0625	11.16	Plastic	Grouted	3 3/8	0.477	456.1	28%
Set8: P7	24	7.125	11.16	Plastic	Grouted	3	0.421	535.5	32%
Set9: P2	23.9375	7.125	10.19	Contr		0	0.000	1475.0	93%
Set9: P3	24.0625	7.0625	10.19	Plastic	Grouted	3	0.424	548.7	34%
Set11: P1	24	9.25	9.25	Contr		0	0	1354.6	96%
Set11: P2	24.125	9.1875	9.25	Plastic	Grouted	3.375	0.367	528.2	38%
Set11: P3	24.125	9.25	9.25	Steel	Grouted	3.375	0.364	750.3	53%
Set11: P4	24	7.25	9.25	Contr		0	0	1461.5	100%
Set11: P6	23.875	7.25	9.25	Steel	Grouted	3	0.414	785.1	54%
Set11: P7	24.125	5.125	9.25	Contr		0	0.000	842.0	100%
Set11: P8	24.125	5.25	9.25	Plastic	Grouted	2.375	0.452	518.2	62%
Set11: P9	24.1875	5.1875	9.25	Steel	Grouted	2.375	0.457	709.5	84%